List of Figures

Chapter Thirty-two

BRIDGE HYDRAULICS

32-1.0 INTRODUCTION

32-1.01 Definition

Bridges are defined as follows:

1. structures that transport traffic over waterways, railroads, roads or other obstructions;

2. part of a stream crossing system that includes the approach roadway over the floodplain, relief openings and the bridge structure; and

3. legally, structures with a total span of 6.1 m or more measured along the centerline of the roadway. For multiple-pipe structures, this includes the distance between the pipes. However, structures designed hydraulically as bridges as described above are treated in this Chapter, regardless of length.

See Figure 31-1A, Maximum Span Lengths for Culverts, for precise definitions on the measurement of span length to distinguish between a bridge and a culvert.

32-1.02 Analysis/Design

Proper hydraulic analysis and design is as vital as the structural design. Stream crossing systems should be designed for the following:

1. minimum cost subject to applicable criteria;
2. desired level of hydraulic performance up to an acceptable risk level;
3. mitigation of impacts on stream environment; and
4. accomplishment of social, economic and environmental goals.

32-1.03 Purpose of Chapter

This Chapter includes the following:

1. guidance in the hydraulic design of a stream crossing system through the following:
   a. appropriate policy and design criteria, and
   b. technical aspects of hydraulic design;
2. non-hydraulic factors that influence design including the following:
   a. environmental concerns,
   b. emergency access, traffic service, and
   c. consequences of catastrophic loss;
3. a design procedure which emphasizes hydraulic analysis using the computer programs WSPRO and HEC-2; and
4. a brief section on design philosophy. A more in-depth discussion is presented in the AASHTO *Highway Drainage Guidelines*, Chapter VII.

32-2.0 POLICY

32-2.01 General Policy

Policy is a set of goals and/or a plan of action. Federal policies and State policies that broadly apply to drainage design are presented in Chapter Twenty-eight. Policies that are unique to bridge crossings are presented in this Section.

The hydraulic analysis should consider various stream crossing system designs to determine the most cost-effective proposal consistent with design constraints.

32-2.02 General INDOT Policies

These general INDOT policies identify specific areas for which quantifiable criteria can be developed.

1. The final design selection should consider the maximum backwater allowed by IDNR or INDOT. See Chapter Twenty-nine.

2. The final design should not significantly alter the flow distribution in the floodplain.

3. The "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge.

4. A 600-mm freeboard should be established to allow for passage of ice and debris. For navigation channels, a vertical clearance conforming to Federal and/or IDNR requirements should be established based on normally expected flows during the navigation season.

5. Degradation or aggradation of the river and contraction/local scour shall be estimated, and appropriate positioning of the foundation, below the total scour depth if practical, shall be included as part of the final design.

32-2.03 INDOT Bridge Sizing Policy

The INDOT bridge sizing policy is segregated into two main categories.

1. Category 1. Bridges that do not require an Indiana Department of Natural Resources (IDNR) Permit, and

2. Category 2. Bridges that require an IDNR Permit.

Projects that require an IDNR Permit (Certificate of Approval for Construction in a Floodway) are the following:

1. structures with a drainage area greater than 130 km^2 in a rural area, or
2. structures with a drainage area greater than 2.6 km^2 in an urban area.

"Rural area" as defined in the Indiana Register, Volume 16, Number 6, March 1, 1993, p. 1527 means the following:

> *An area where the flood protection grade of each residential, commercial, or industrial building impacted by this project is higher than the regulatory flood elevation under the project control, and where the area lies outside:*

(i) *the corporate boundaries of the consolidated city or an incorporated city or town; and*

(ii) *the territorial authority for comprehensive planning established under IC 36-7-4-205(b).*

Basically, an area cannot be rural if it lies within a city or its planning zone or if any finished floor elevation within the project backwater limits is below the Q_{100} elevation.

Projects with a drainage area less than those listed above will not require a permit from IDNR. See Chapter Nine for more information on the IDNR Construction in a Floodway Permit.

For new bridges on new alignment, the maximum backwater shall not exceed 40 mm. The 40-mm maximum may be modified for the following special cases.

1. the backwater dissipates to 40 mm or less at the right-of-way line, or

2. the channel is sufficiently deep to contain the increased elevation without overtopping the banks, or

3. flood easements may be purchased upstream of the bridge to allow for greater than 40 mm of backwater. The cost savings should be great enough to offset the cost of the flood easements and the possible delay in constructing the project. Past experience has shown that this option may delay a project by one or more years.

The Hydraulics Engineer must approve any exceptions to the 40-mm backwater allowance for new bridges on new alignment.

For existing (or "baseline") conditions, the IDNR essentially limits surcharge to 30 mm (urban and rural). Existing conditions are defined as the water surface profile that would result from only those encroachments that have been in place since 1973. Although IDNR policy will allow for a slight increase over existing conditions, INDOT will not. INDOT policy for bridge replacements and bridge rehabilitations is that the surcharge created by a proposed structure must be equal to or less than the existing surcharge, unless the existing surcharge is less than 30 mm. This will allow future widening of the structure. In addition, if the surcharge created by an existing structure is greater than 300 mm, the proposed surcharge for the bridge replacement or bridge rehabilitation project must be no greater than 300 mm above the natural channel flood profile.

FHWA does not require economic justification for bridges that cause less than 300 mm of backwater. Therefore, a formal risk assessment will not be required in these cases.

32-3.0 DESIGN CRITERIA

Design criteria are the tangible means for placing accepted policies into action and become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change when conditions so dictate as approved by the Hydraulics Engineer.

32-3.01 General Criteria

Following are the AASHTO general criteria related to the hydraulic analyses for the location and design of bridges as stated in the *Highway Drainage Guidelines*.

1. Backwater will not significantly increase flood damage to property upstream of the crossing.

2. Velocities through the structure(s) will not damage the highway facility nor increase damages to adjacent property.

3. Maintain the existing flow distribution as practical.

4. Provide pier spacing and orientation and abutments designed to minimize flow disruption and potential scour.

5. Provide foundation design and/or scour countermeasures to avoid failure by scour.

6. Design pier spacing and freeboard at structure(s) to pass anticipated debris and ice.

7. Consider acceptable risks of damage or viable measures to counter the vagaries of alluvial streams.

8. Consider minimal disruption of ecosystems and values unique to the floodplain and stream.

9. Provide a level of traffic service compatible with that commonly expected for the class of highway and compatible with projected traffic volumes.

10. Design choices should support costs for construction, maintenance and operation, including probable repair and reconstruction and potential liability, that are affordable.

32-3.02 INDOT Criteria

The criteria in this Section and Figure 32-3A, Design Storm Frequency (Bridge Waterway Openings), augment the general criteria in Section 32-3.01. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using the water surface profile programs WSPRO or HEC-2.

32-3.02(01) Roadway Serviceability

The traveled way overtopping flood level identifies the limit of serviceability.

32-3.02(02) Design Floods

Structures and their approach roadways shall at a minimum be designed for the passage of the design year flood specified for the required road serviceability for the highway classification system specified. By definition, design floods will not overtop the roadway. As noted in Chapter Twenty-nine and Figure 32-3A, Design Storm Frequency (Bridge Waterway Openings), the backwater must be calculated from the Q_{100} flood which, in some cases, may exceed the design flood.

32-3.02(03) Freeboard

Where practical, a minimum clearance of 600 mm shall be provided between the design approach water surface elevation and the low chord of the bridge for the final design alternative to allow for passage of ice and debris. Where this is not practical, the clearance should be established by the designer based on the type of stream and level of protection desired as approved by INDOT. For example, 300 mm should be adequate on small streams that normally do not transport drift. Urban bridges with grade limitations may not provide any freeboard. A 1.0-m freeboard is desirable on major rivers which are known to carry large debris. The crest vertical curve profile is the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge.

32-3.02(04) Span Lengths

The minimum span length for bridges having more than 3 spans should be 30 m for those spans over the main channel. Three-span bridges shall have the center span length maximized at sites where debris may be a problem. For two-span bridges, span lengths must be approved by the Hydraulics Engineer.

32-3.02(05) Flow Distribution

The conveyance of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility shall not cause any significant change in the existing flow distribution.

32-3.02(06) Scour

Design for bridge foundation scour considering the magnitude of flood, including the 100-year (1%) event, that generates the maximum scour depth. The design shall use a geotechnical design practice safety factor of from 2 to 3. The resulting design should then be checked using a super flood (Q_{500}). Use 1.7 times the magnitude of the 100-year (1%) event if no other source for Q_{500} is available and a geotechnical design practice safety factor of at least 1.0. See Section 32-6.08.

32-3.02(07) Temporary Runaround Structures

Temporary runaround structures are usually operational for three months to two years. Therefore, the serviceability criteria is greatly reduced. At a minimum, a temporary runaround shall be serviceable during a Q_2 discharge and shall be checked for allowable backwater at a Q_{100} discharge. The best way to achieve this objective is to set the grade of the temporary runaround as close as possible to the elevations corresponding with the serviceability levels in Figure 32-3B, Design Storm Frequency for Temporary Structures.

For all facility types, backwater must be computed for the design storm and Q_{100} discharge. For structures requiring an IDNR permit, the backwater at Q_{100} must not exceed 40 mm over the base condition (i.e., existing backwater elevation). "Base condition" means the condition of the flood plain on January 1, 1973, but without an unauthorized dam or levee. If an activity after December 31, 1972 lowered the regulatory flood profile, the flood plain under the lower profile is the base condition. For structures not requiring an IDNR permit, the backwater from the Q_{100} shall not exceed the finished floor elevations of nearby buildings or residences.

32-4.0 DESIGN PROCEDURE

32-4.01 Survey Accuracy (Computation Method)

The design for a stream crossing system requires a comprehensive engineering approach that includes formulation of alternatives, data collection, selection of the most cost-effective alternative according to established criteria and documentation of the final design.

Water surface profiles are computed for a variety of technical uses including the following:

1. flood insurance studies,
2. flood hazard mitigation investigations,
3. drainage crossing analysis, and
4. longitudinal encroachments.

The completed profile can affect the highway bridge design and is the mechanism for determining the effect of a bridge opening on upstream water levels. Errors associated with computing water surface profiles with the step-backwater profile method can be classified as follows:

1. data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation;

2. errors in accuracy of energy loss calculations depending on the validity of the energy loss equation employed and the accuracy of the energy loss coefficients (Manning's n-value is the coefficient measuring boundary friction);

3. inadequate length of stream reach investigated; and

4. significant computational errors resulting from using cross sectional spacings which are incorrectly considered to be adequate. The errors are due to inaccurate integration of the energy loss-distance relationship that is the basis for profile computations. These errors may be reduced by adding interpolated or actual sections (more calculation steps).

32-4.02 Design Procedure Outline

The following design procedure outline shall be used. Although the scope of the project and individual site characteristics make each design unique, this procedure shall be applied unless indicated otherwise by INDOT.

I. DATA COLLECTION
   A. Survey
      1. Topography
      2. Geology
      3. High-water marks
      4. History of debris accumulation, ice and scour
      5. Maps, aerial photographs
      6. Field reconnaissance
   B. Influences On Hydraulic Performance of Site
      1. Other streams, reservoirs, water intakes
      2. Structures upstream or downstream
      3. Natural features of stream and floodplain
      4. Channel modifications upstream or downstream
      5. Floodplain encroachments
      6. Sediment types and bed forms (Also see Appendix C, Scour, Site Data, Level I Qualitative Analysis, FHWA HEC 20, 1991)
   C. Environmental Impact
      1. Existing bed or bank instability (Level I)
      2. Floodplain land use and flow distribution
      3. Environmentally sensitive areas (fisheries, wetlands, etc.)
      4. Level I Qualitative Analysis (FHWA HEC 20, 1991)
   D. Site-Specific Design Criteria
      1. Road serviceability (design frequency)
      2. Flood damage potential
      3. Freeboard

II. HYDROLOGIC ANALYSIS
   A. Studies By Other Agencies
      1. Federal Flood Insurance Studies
      2. Federal Floodplain Studies by the COE, NRCS, etc.
      3. IDNR and Local Floodplain Studies
      4. Hydraulic performance of existing bridges
   B. Watershed Morphology
      1. Drainage area (attach map)
      2. Watershed and stream slope
      3. Channel geometry

C. Hydrologic Computations
   1. Discharge and frequency for historical flood that complements the high-water marks used for calibration
   2. Discharges for specified frequencies

III. HYDRAULIC ANALYSIS
   A. Computer Model Calibration and Verification
   B. Hydraulic Performance for Existing Conditions
   C. Hydraulic Performance of Proposed Designs
   D. Scour Computations

IV. SELECTION OF FINAL DESIGN
   A. Measure of compliance with established hydraulic criteria
   B. Compare proposed bridge size and backwater to the existing bridge
   C. Consider environmental and social criteria
   D. Make final selection
   E. Design details such as riprap, scour abatement, river training, etc.

V. DOCUMENTATION

A bridge waterway study is required at the Hydraulics Grade Review plan submittal. This study requires the following:

A. <u>Drainage Area Determination</u>. Drainage areas can be obtained by planimeter from USGS Quad Maps using the USGS manual *Drainage Areas of Indiana Streams* for supplemental data as needed. On large streams, drainage areas can be obtained from the USGS Manual *Drainage Areas of Indiana Streams*. The sources used must be photocopied and attached to the bridge waterway study.

B. <u>Hydrologic Analysis</u>. Complete documentation of the method used and all relevant variables. See Chapter Twenty-nine for allowable methods.

C. <u>Hydraulic Analysis of Existing and Proposed Conditions</u>. See Section 32-4.04. A hard copy of all input and output files and a site plan that illustrates the location of all cross sections used in the hydraulic analysis is required. A diskette with all input files must also be submitted.

D. <u>Justification of Selected Bridge</u>. A summary of the results of the hydraulic analysis of various bridge types is required.

E. <u>Summary of Hydraulic Parameters</u>. Summarize the bridge waterway study by listing the following parameters:

1. drainage area
2. Q_{100} discharge
3. Q_{100} elevation
4. backwater
5. velocity
6. waterway area
7. low-structure elevation
8. skew
9. existing waterway opening
10. existing low-structure elevation
11. existing backwater

F. Files. Provide one diskette containing all input and output files for WSPRO and/or HEC-2.

G. Layout Sheet (LPA Projects Only). This must show the following:
1. profile grade (existing and proposed)
2. waterway opening
3. pier placement
4. superstructure
5. hydraulic data
6. Q_{100} elevation

If road overflow is expected, the Plan and Profile Sheet must show the limits of road overflow. A checklist form is presented in Section 32-7.0.

32-4.03 Hydraulic Performance of Bridges

The stream crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using WSPRO or HEC-2 unless indicated otherwise by INDOT. Alternative methods of analysis of bridge hydraulics are discussed in this Section but emphasis is placed on the use of WSPRO.

It is impractical to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations. However, an example of the basic manual calculations is included in the AASHTO *Model Drainage Manual*, Chapter 10, Appendix D, as an explanation of the various aspects of bridge hydraulics.

The hydraulic variables and flow types are defined in Figure 32-4A, Bridge Hydraulics Definitions Sketch, and Figure 32-4B, Bridge Flow Types. The following applies.

1. Backwater (h1) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross section (Section 1). It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 32-4B.

2. Type I consists of subcritical flow throughout the approach, bridge and exit cross sections and is the most common condition encountered in practice.

3. Types IIA and IIB both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In Type IIB, it is higher than the normal water surface elevation, and a weak hydraulic jump immediately downstream of the bridge contraction is possible.

4. Type III flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

32-4.04 Methodologies

No single method is ideally suited for all situations. If a satisfactory computation cannot be achieved with a given method, an alternative method should be attempted. However, it has been found that, with careful attention to the setup requirements of each method, essentially duplicative results can usually be achieved using both momentum and energy methods.

32-4.04(01) Momentum

The Corps of Engineers' HEC-2 model uses a variation of the momentum method in the special bridge routine where there are bridge piers. The momentum equation between cross sections 1 and 3 is used to detect Type II flow and solve for the upstream depth in this case with critical depth in the bridge contraction.

This model has been used for the majority of the flood insurance studies performed under the NFIP. However, some believe that the bridge analysis routines in HDS-1 and WSPRO may yield a better definition of actual hydraulic performance.

32-4.04(02) HEC-RAS

The Corps of Engineers' Hydrologic Engineering Center (HEC) has developed the HEC-RAS (River Analysis System) program package. It operates under WINDOWS and has full graphic support. The package includes all the features inherent to HEC-2 and WSPRO plus program selected friction slope methods, mixed flow regime capability, automatic "n" value calibration, ice cover, quasi 2-D velocity distribution, superelevation around bends, bank erosion, riprap design, stable channel design, sediment transport calculations and scour at bridges

HEC-RAS Version 2.2 will provide the outputs necessary to evaluate bridge hydraulics for all flow types. However, earlier versions will not be accepted for bridge hydraulic computations.

32-4.04(03) Energy (HDS-1)

The method developed by FHWA described in HDS-1 is an energy approach with the energy equation written between cross sections 1 and 4 as shown in Figure 32-4B, Bridge Flow Types, for Type I flow. The backwater is defined in this case as the increase in the approach water surface elevation relative to the normal water surface elevation without the bridge.

This model utilizes a single typical cross section to represent the stream reach from points 1 to 4 on Figure 32-4B. It also requires the use of a single energy gradient. This method is no longer allowed by INDOT for final design analysis of bridges due to its inherent limitations, but it may be useful for preliminary analysis and training. Studies performed by the Corps of Engineers for the FHWA show the need to utilize a multiple cross-section method of analysis to achieve reasonable stage-discharge relationships at a bridge.

32-4.04(04) Energy (WSPRO)

WSPRO combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping and flow through multiple openings and culverts. It also includes an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary when approach flows occur on very wide, heavily vegetated floodplains. The program also greatly facilitates the hydraulic analysis required to determine the least-cost alternative.

WSPRO is suggested for both preliminary and final analyses of bridge hydraulics. Even if only a single surveyed cross section is available, the input-data propagation features of WSPRO make it easy to apply with more comprehensive output available than with HDS-1.

32-4.04(05) Two-Dimensional Modeling

The water surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analysis is performed using one-dimensional methods such as the standard step method found in WSPRO or HEC-2. Although one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-stream water surface elevations, flow velocities or flow distribution.

Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same field data as a one-dimensional model and, depending on complexity, may require a little more computer time.

“Bri-Stars” is a semi-two-dimensional model capable of computing alluvial scour/deposition through subcritical, supercritical and a combination of both flow conditions involving hydraulic jumps. It is capable of simulating channel widening/narrowing and local scour due to highway encroachments.

It has a bridge component which allows the computation of hydraulic flow variables and the resulting scour. “Bri-Stars” also includes a companion expert system program which allows classifying streams by their morphological properties. See Section 30-5.0 for a more complete discussion of “Bri-Stars.”

The USGS has developed a two-dimensional finite element model for the FHWA that is designated FESWMS. This model has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist. This two-dimensional modeling system is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, floodplain encroachments, multiple channels, flow around islands and flow in estuaries. Where the flow is essentially two-dimensional in the horizontal plane, a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

32-4.04(06) Physical Modeling

Complex hydrodynamic situations defy accurate or practical mathematical modeling. Physical models should be considered as follows:

1. hydraulic performance data are needed that cannot be reliably obtained from mathematical modeling,

2. risk of failure or excessive over-design is unacceptable, and

3. research is needed.

The constraints on physical modeling are as follows:

1. size (scale),
2. cost, and
3. time.

32-4.05 WSPRO Modeling

In theory, the water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the bridge backwater. In practice, all of the cross sections that are actually necessary for the energy analysis through the bridge opening for a single-opening bridge without spur dikes are shown in Figure 32-4C, Cross Section Locations for Stream Crossing with a Single Waterway Opening. The additional cross sections that are necessary for computing the entire profile are not shown in this Figure. Cross sections 1, 3 and 4 are required for a Type I flow analysis and are referred to as the approach section, bridge section and exit section, respectively. In addition, cross section 3F, which is called the full-valley section, is needed for the water surface profile computation without the presence of the bridge contraction. Cross section 2 is used as a control point in Type II flow but requires no input data. Two more cross sections must be defined if spur dikes and a roadway profile are specified.

Pressure flow through the bridge opening is assumed to occur where the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. WSPRO can also simultaneously consider embankment overflow as a weir discharge. This leads to flow classes 1 through 6 as given in Figure 32-4D, Flow Classification According to Submergence Conditions (WSPRO User Instructors Manual - 1990).

In free-surface flow, there is no contact between the water surface and the low-girder elevation of the bridge. In orifice flow, only the upstream girder is submerged, while in submerged orifice flow both the upstream and downstream girders are submerged. A total of four different bridge types can be analyzed.

The user's instruction manual for WSPRO should serve as a source for more detailed information on using the computer model. Some specific example problems are given in Appendix B of Chapter 10 of the *Model Drainage Manual* with sample computer input and output data provided. Only enough information to understand the examples is included.

32-4.06 HEC-2 Modeling

In theory, the water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the bridge backwater. The cross sections that are necessary in practice for the analysis of a single opening bridge using the special bridge option are shown in Figure 32-4E, Cross Section Locations in the Vicinity of Bridges.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Secondly, the loss through the structure itself is computed by either the normal bridge or the special bridge methods.

The user's instructional manual for HEC-2 should serve as a source for more detailed information for using the computer model. Numerous input/output examples are provided. HEC-2 has its own data creation package (COED) that assists the user with preparing/editing input data and includes online help features. A separate, stand-alone data editing program which checks for input/modeling errors is also provided. Cross section, water surface profile and rating curve viewing/plotting/ printing are provided using the PLOT2 program.

The normal bridge method handles the cross section at the bridge just as it would any river cross section with the exception that the area of the bridge below the water surface is subtracted from the total area and the wetted perimeter is increased where the water surface elevation exceeds the low chord. The normal bridge method is particularly applicable for bridges without piers, bridges under high submergence, and for low flow through circular and arch culverts. Wherever flow crosses critical depth in a structure, the special bridge method should be used. The normal bridge method is automatically used by the computer, even though data were prepared for the special bridge method, for bridges without piers and under low flow control.

The special bridge method can be used for any bridge, but it should be used for bridges with piers where low flow controls, for pressure flow, and wherever flow passes through critical depth when passing through the structure. The special bridge method computes losses through the structure for low flow, weir flow and pressure flow or for any combination of these.

A series of program capabilities are available to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flows to leveed channels, to block out road fills and bridge decks, and to analyze floodplain encroachments.

Cross sections with low overbank areas or levees require special consideration in computing water surface profiles because of possible overflow into areas outside the main channel. Normally the computations are based on the assumption that all area below the water surface elevation is effective in passing the discharge. However, if the water surface elevation at a particular cross section is less than the top of levee elevations, and if the water cannot enter or leave the overbanks upstream of that cross section, then the flow areas in these overbanks should not be used in the computations. Variable IEARA on the X3 card and the bank stations coded in fields three and four on the X1 card are used for this condition. By setting IEARA equal to ten, the program will consider only flow confined by the levees, unless the water surface elevation is above the top of one or both of the levees, in which case flow area or areas outside the levee(s) will be included. If this option is employed and the water surface elevation is close to the top of a levee, it may not be possible to balance the assumed and computed water surface elevations due to the changing assumptions of flow area where just above and below the levee top. Where this condition occurs, a note will be printed that states that the assumed and computed water surface elevations for the cross section cannot be balanced. A water surface elevation equal to the elevation which came closest to balancing will be adopted. It is then up to the program user to determine the appropriateness of the assumed water surface elevation and start the computation over again at that cross section if required.

It is important for the user to study carefully the flow pattern of the river where levees exist. If, for example, a levee were open at both ends and flow passed behind the levee without overtopping it, IEARA equals zero or blank should be used. Also, assumptions regarding effective flow areas may change with changes in flow magnitude. Where cross section elevations outside the levee are considerably lower than the channel bottom, it may be necessary to set IEARA equal to ten to confine the flow to the channel.

A user’s instruction manual for HEC-2 is available and should serve as a source for more detailed information on using this computer model. Specific examples are given in Appendix C of Chapter 10 of the *Model Drainage Manual*.

32-5.0 BRIDGE SCOUR OR AGGRADATION

32-5.01 Introduction

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis. The hydraulics engineer must endeavor to always be aware of and use the most current scour forecasting technology.

The FHWA issued a Technical Advisory (TA 5140.20) on bridge scour in September 1988. The document "Interim Procedures for Evaluating Scour at Bridges" was an attachment to the Technical Advisory. The interim procedures were replaced by HEC 18 (1991, 1993, 1995). Users of this *Manual* should consult the latest version HEC 18 for a more thorough treatise on scour and scour prediction methodology. A companion FHWA document to HEC 18 is HEC 20 *Stream Stability at Highway Structures*.

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analyses using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations. This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in HEC 20.

Less hazardous, perhaps, are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. Also, aggradation in a stream reach may serve to moderate potential scour depths. Aggradation is sometimes referred to as negative scour.

32-5.02 Scour Types

Present technology dictates that bridge scour be evaluated as the following interrelated components.

1. long-term profile changes (aggradation/degradation),
2. plan form change (lateral channel movement),
3. contraction scour/deposition, and
4. local scour.

32-5.02(01) Long-Term Profile Changes

Long-term profile changes can result from stream bed profile changes that occur from aggradation and/or degradation.

1. Aggradation is the deposition of bedload due to a decrease in the energy gradient.

2. Degradation is the scouring of bed material due to increased stream sediment transport capacity which results from an increase in the energy gradient.

Forms of degradation and aggradation should be considered as imposing a permanent future change for the stream bed elevation at a bridge site whenever they can be identified.

32-5.02(02) Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches and increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

32-5.02(03) Contraction

Channel contraction scour results from a constriction of the channel which may, in part, be caused by bridge piers in the waterway. Deposition results from an expansion of the channel or the bridge site being positioned immediately downstream of a steeper reach of stream. Highways, bridges and natural channel contractions are the most commonly encountered cause of contraction scour. Two practices are provided in this *Manual* for estimating deposition or contraction scour:

1. Sediment routing practice. This practice should be considered if either bed armoring or aggradation from an expanding reach is expected to cause an unacceptable hazard.

2. Empirical practice. This practice is adapted from laboratory investigations of bridge contractions in non-armoring soils and, as such, must be used considering this qualification. This practice does not consider bed armoring and its application for aggradation may be technically weak.

The same empirical practice algorithms used in this *Manual* to evaluate a naturally contracting reach may also be used to evaluate deposition in an expanding reach provided armoring is not expected to occur. With deposition the practice of applying the empirical equations "in reverse" is required; i.e., the narrower cross section is upstream which results in the need to manipulate the use of the empirical "contraction scour" equation. This need to manipulate the intended use of an equation does not occur with the sediment routing practice which is why it may be more reliable in an expanding reach.

32-5.02(04) Local Scour

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood-flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry. However, the importance of these geometric variables will vary. As an example, increasing the pier or cofferdam width either through design or debris accumulation will increase the amount of local scour, but only up to a point in subcritical flow streams. After reaching this point, pier scour should not be expected to measurably increase with increased stream velocity or depth. This threshold has not been defined in the more rare, supercritical flowing streams.

When fendering or other pier protection systems are used, their effect on pier scour and collection of debris shall be considered in design. The stability of abutments in areas of turbulent flow shall be thoroughly investigated, and exposed embankment slopes should be protected with appropriate scour countermeasures.

32-5.03 Armoring

Armoring occurs because a stream or river is unable, during a particular flood, to move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit to form a layer of riprap-like armor on the stream bed or in the scour holes and thus limit further scour for a particular discharge. This armoring effect can decrease scour hole depths which were predicted based on formulae developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring may also cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, difficult-to-assess plan form changes. Bank widening also spreads the approach flow distribution which in turn results in a more severe bridge opening contraction.

32-5.04 Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, this material is replaced with more easily scoured material. Thus, at some later date another flood may reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour resistant, such as consolidated soils and glacial till, and so-called bed rock streams and streams with gravel and boulder beds.

32-5.05 Scour Analysis Methods

Before the various scour forecasting methods for contraction and local scour can be applied, it is first necessary to (l) obtain the fixed bed channel hydraulics, (2) estimate the profile and plan form scour or aggradation, (3) adjust the fixed bed hydraulics to reflect these changes, and (4) compute the bridge hydraulics. Two methods are provided in the AASHTO *Model Drainage Manual*, Chapter 10, for combining the contraction and local scour components to obtain total scour. The first method, Method 1, has application where armoring is not a concern or insufficient information is available to permit its evaluation, or where more precise scour estimates are not deemed necessary. The second method identified as Method 2 may be used where stream bed armoring is of concern, more precise contraction scour estimates are deemed necessary or deposition is expected and is a primary concern. INDOT typically uses Method 1, which is described in the following.

Method 1 is considered a conservative practice because it assumes that the scour components develop independently. Thus, the potential local scour to be calculated using this Method would be added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The general approach with Method 1 is as follows:

1. Estimate the natural channel's hydraulics for a fixed bed condition based on existing conditions.

2. Assess the expected profile and plan form changes.

3. Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.

4. Estimate contraction scour using the empirical contraction formula and the adjusted fixed bed hydraulics assuming no bed armoring.

5. Estimate local scour using the adjusted fixed bed channel and bridge hydraulics assuming no bed armoring.

6. Add the local scour to the contraction scour to obtain the total scour. If contraction scour is negative, then use zero for contraction scour.

32-5.06 Scour Assessment Procedure

Bridge scour assessment shall normally be accomplished by collecting the data and applying the general procedure outlined in this Section. An example problem demonstrating the scour computations is included in the AASHTO *Model Drainage Manual*, Chapter 10, Appendix B.

32-5.06(01) Site Data

1. Bed Material. Obtain bed material samples for all channel cross sections where armoring will be evaluated. If armoring is not being evaluated, this information need only be obtained at the site. From these samples try to identify historical scour and associate it with a discharge. Also, determine the bed material size distribution in the bridge reach and from this distribution determine d16, d50, d84, and d90. This will be accomplished by using the appropriate soil boring(s).

2. Geometry. Obtain existing stream and floodplain cross sections, stream profile, site plan and the stream's present, and where possible, historic geomorphic plan form. Also, locate the bridge site with respect to other bridges in the area, tributaries to the stream or close to the site, bed rock controls, manmade controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any "headcuts" due to natural causes or, for example, gravel mining operations. Upstream gravel mining operations may absorb the bed material discharge resulting in the more adverse clear water scour case discussed later. Any data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

3. Historic Scour. Obtain any scour data on other bridges or similar facilities along the stream.

4. Hydrology. Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent as well as whether it is "flashy" or subject to broad hydrograph peaks resulting from gradual flow increases such as occur with general thunderstorms or snowmelt.

5. Geomorphology. Classify the geomorphology of the site; i.e., whether it is a floodplain stream, crosses a delta or crosses an alluvial fan; youthful, mature or old age.

32-5.06(02) General

1. Step 1. Decide which analysis method is applicable. Method 1 shall be used to evaluate existing bridges to identify significant potential scour hazards or, where armoring is obviously not of concern, on a proposed bridge. Method 2 should be used to evaluate bridges where significant armoring may occur.

2. Step 2. Determine the magnitude of the 100-year flood and the 500-year super flood.

3. Step 3. Develop a water surface profile through the site's reach for fixed bed conditions using WSPRO or HEC-2.

4. Step 4. Obtain the variables necessary to perform contraction and local scour.

5. Step 5. Compute the predicted scour depths using the equations in HEC 18 for contraction and pier scour for the 100-year and 500-year floods or an overtopping flood of a lesser recurrence interval.

6. Step 6. Once an acceptable scour threshold is determined, the geotechnical engineer can prepare a foundation recommendation for the bridge based on the scour information obtained from the foregoing procedure and using commonly accepted safety factors. The structural engineer should evaluate the lateral stability of the bridge based on the foregoing scour.

   Spread footings on soil or erodible rock shall be located so that the bottom of footing is below scour depths determined for the check flood for scour. Spread footings on scour-resistant rock shall be designed and constructed to maintain the integrity of the supporting rock.

7. Step 7. Repeat the foregoing assessment procedures using the greatest bridge opening flood discharge associated with the selected 500-year “super flood” or an overtopping flood of a lesser recurrence interval. These findings are again for the geotechnical engineer to use in evaluating the foundation recommendation obtained in Step 6. A foundation design safety factor of 1.0 is commonly used to ensure that the bridge is marginally stable for a flood associated with the 500-year “super flood”.

32-5.07 Pressure Flow Scour

The following information is extracted from the FHWA Publication HEC 18 *Evaluating Scour at Bridges*, April, 1993.

Pressure flow, which is also denoted as orifice flow, occurs where the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. Pressure flow under the bridge results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subject to pressure flow results from the flow being directed downwards toward the bed by the superstructure and by increasing the intensity of the horse shoe vortex. The vertical contraction of the flow is a more significant cause of the increase in scour depth. However, in many cases, where a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of discharge which must pass under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to over-topping.

WSPRO or HEC-2 can be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers where flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment will then be exercised to determine the appropriate multiplier times the calculated pier scour depth for the pressure flow scour depth. This multiplier ranges from 1.0 for low approach froude numbers (Fr = 0.1) to 1.6 for high approach froude numbers (Fr = 0.6). If the bridge is overtopped, the depth to be used in the pier scour equations and for computing the froude number is the depth to the top of the bridge deck or guardrail obstructing the flow.

32-6.0 DESIGN PHILOSOPHY

32-6.01 Introduction

Any stream is a dynamic natural system which, as a result of the encroachment caused by elements of a stream-crossing system, will respond in a way that may well challenge even an experienced hydraulics engineer. The complexities of the stream response to encroachment demand that (l) hydraulics engineers must be involved from the outset in the choice of alternative stream crossing locations, and (2) at least some of the members of the engineering design team must have extensive experience in the hydraulic design of stream-crossing systems. Hydraulics engineers should also be involved in the solution of stream stability problems at existing structures.

This Section discusses qualitatively some of the design issues which contribute to the overall complexity of spanning a stream with a stream-crossing system. A much more thorough discussion of design philosophy and design considerations is found in the AASHTO *Highway Drainage Guidelines* "Hydraulic Analyses for the Location and Design of Bridges."

32-6.02 Location of Stream Crossing

Although many factors, including nontechnical ones, enter into the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include floodplain width and roughness, flow distribution and direction, stream type (braided, straight or meandering), stream regime (aggrading, degrading or equilibrium) and stream controls. The hydraulics of a proposed location also affect environmental considerations such as aquatic life, wetlands, sedimentation and stream stability.

Finally, the hydraulics of a particular site determine whether or not certain national objectives such as wise use of floodplains, reduction of flooding losses, and preservation of wetlands can be met.

32-6.03 Coordination, Permits, Approvals

The interests of other government agencies must be considered in the evaluation of a proposed stream-crossing system, and cooperation and coordination with these agencies, especially water resources planning agencies, must be undertaken.

Designers of stream-crossing systems must be cognizant of relevant local, State and Federal laws and permit requirements. Federal and State permits are required for construction of bridges over navigable waters and are issued by the U.S. Coast Guard and IDNR. Permits for other construction activities in navigable waters are under the jurisdiction of the U.S. Army Corps of Engineers. Applications for Federal permits may require environmental impact assessments under the National Environmental Policy Act of 1969. IDNR issues construction in a floodway and lake preservation permits. IDEM issues 401 permits to go with 404 permits when required. If over two hectares of land are disturbed, a Rule 5 (NPDES) permit will be required. See Chapter Nine for more information on permits and certifications.

32-6.04 Environmental Considerations

Environmental criteria which must be met in the design of stream-crossing systems include the preservation of wetlands and protection of aquatic habitat. Such considerations often require the expertise of a biologist on the design team. As a practical matter with bridges, the hydraulic design criteria related to scour, degradation, aggradation, flow velocities and lateral distribution of flow, for example, are important criteria for evaluation of environmental impacts and the safety of the stream-crossing structures.

32-6.05 Stream Morphology

The form and shape of the stream path created by its erosion and deposition characteristics comprise its morphology. A stream can be braided, straight or meandering, or it can be in the process of changing from one form to another as a result of natural or manmade influences. A historical study of the stream morphology at a proposed stream-crossing site is mandatory. (FHWA HEC 20 Level I Analysis) This study shall also include an assessment of any long-term trends in aggradation or degradation. Braided streams and alluvial fans shall especially be avoided for stream-crossing sites whenever possible.

32-6.06 Data Collection

The purpose of data collection is to gather all necessary site information. This shall include such information as topography and other physical features, land use and culture, flood data, basin characteristics precipitation data, historical high-water marks, existing structures, channel characteristics and environmental data. A site plan shall be developed on which much of the data can be shown.

32-6.07 Scour

The extreme hazard posed by bridges subject to bridge scour failures dictates a different philosophy in selecting suitable flood magnitudes to use in the scour analysis. With bridge flood hazards other than scour, such as those caused by roadway overtopping or property damage from inundation, a prudent and reasonable practice is to first select a design flood to determine a trial bridge opening geometry. This geometry is either subjectively or objectively selected based on the initial cost of the bridge along with the potential future costs for flood hazards. Following the selection of this trial bridge geometry, the base flood (100-year) is used to evaluate this selected opening. This two-step evaluation process is used to ensure the selected bridge opening based on the design flood contains no unexpected increase in any existing flood hazards other than those from scour or aggradation. With bridge scour, not only is it required to consider bridge scour or aggradation from the base floods (100-year) but, also, an even larger flood termed herein as the “super flood” (500-year).

Scour prediction technology is steadily developing but currently lacks the reliability associated with other facets of hydraulic engineering. Several formulae for predicting scour depths are currently available and others will certainly be developed in the future. The designer should strive to be acquainted with the “state of practice” at the time of a given analysis and is encouraged to be conservative in the resulting scour predictions.

First discussion is warranted as to what constitutes the greatest discharge passing through the bridge opening during a particular flood. Even where there are relief structures on the floodplain or overtopping occurs, some flood other than the base flood or “super flood” may cause the worst case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a floodplain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings may not result in reduction in the bridge opening discharge.

With potential bridge scour hazards a different flood selection and analysis philosophy is considered reasonable and prudent. The foregoing trial bridge opening which was selected by considering initial costs and future flood hazard costs shall be evaluated for two possible scour conditions with the worse case dictating the foundation design - and possibly a change in the selected trial bridge opening.

First, evaluate the proposed bridge and road geometry for scour using the base flood. Once the expected scour geometry has been assessed, the geotechnical engineer would recommend the foundation. This foundation design would use the conventional foundation safety factors and eliminate consideration of any stream bed and bank material displaced by scour for foundation support.

Second, impose a "super flood" on the proposed bridge and road geometry. This event shall be greater than the base flood and shall be used to evaluate the proposed bridge opening to ensure that the resulting potential scour will produce no unexpected scour hazards. The Department's "super flood" shall be defined as the 500-year flood or a designated ratio (e.g., 1.7) times the 100-year flood. Similar to the base flood, evaluate the selected bridge opening using the "super flood." The foundation design based on the base flood would then be reviewed by the geotechnical engineer using a safety factor of 1.0 and again, considering stream bed and bank material displaced by scour from the "super flood."

32-6.08 Preventive/Protection Measures

Based on an assessment of potential scour provided by the Hydraulics Engineer, the structural designers can incorporate design features that will prevent or mitigate scour damage at piers. In general, circular piers or elongated piers with circular noses and an alignment parallel to the flood flow direction are a possible alternative. Spread footings should be used only where the stream bed is extremely stable below the footing and where the spread footing is founded at a depth below the maximum scour computed in Section 32-6.07. Footings may be founded above the scour elevation when they are keyed into non-erodible rock. Drilled shafts or drilled piers are possible where pilings cannot be driven. Protection against general stream bed degradation can be provided by drop structures or grade-control structures in, or downstream of, the bridge opening.

Rock riprap is often used, where stone of sufficient size is available, to armor abutment fill slopes and the area around the base of piers. Riprap design information is presented in Chapter Thirty-eight.

Wherever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided. Embankment overtopping may be incorporated into the design but should be located well away from the bridge abutments and superstructure. Spur dikes are recommended to align the approach flow with the bridge opening and to prevent scour around the abutments. They are usually elliptically shaped with a major to minor axis ratio of 2.5 to 1. Some agencies have found that a length of approximately 46 m provides a satisfactory standard design. Their length can be determined according to HDS 1. Spur dikes, embankments and abutments shall be protected by rock riprap with a filter blanket or other revetments approved by INDOT.

32-6.09 Deck Drainage

A major responsibility of the engineer is to provide for the safety of the traveling public. There is a much greater risk of someone being injured or killed in an accident on the bridge as the result of wet pavement than there is of injury or death due to the catastrophic collapse of the bridge due to floods or structural failure.

Improperly drained bridge decks can cause numerous problems including corrosion, icing and hydroplaning. Wherever possible, bridge decks should be watertight and all deck drainage should be carried to the ends of the bridge. Drains at the end of the bridge should have sufficient inlet capacity to carry all bridge deck drainage.

The design of pavement drainage on the bridge should use the same criteria as the approach roadway. However, it should be noted that an approach roadway with a rural typical section will be more free draining than a bridge deck with parapets where the deck will confine the runoff in a manner similar to a curbed roadway section. Careful attention must be given to spread on the bridge deck.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors shall conform to the procedures presented in Chapter Thirty-three.

Where deck drainage interceptors are needed, a collection system will be necessary to discharge the runoff. Some considerations for this system are as follows:

1. environmental concerns for discharging pavement runoff directly into a waterway,
2. design and maintenance of extensive drain systems attached to the superstructure,
3. free drops from deck interceptors,

4. 150-mm minimum projection beyond lowest adjacent superstructure component, and

5. provide erosion control under free drops unless outlet from bridge superstructure is more than 12.2 m above ground.

32-6.10 Construction Maintenance

Temporary structures and crossings used during construction should be designed for a specified risk of failure due to flooding during the construction period. The impacts on normal water levels and normal flow distribution must be considered.

All borrow areas within the floodplain shall be chosen to minimize the potential for scour and adverse environmental effects within the limits of the bridge and its approaches on the floodplain.

The stream-crossing design shall incorporate measures which reduce maintenance costs whenever possible. These measures include spur dikes, retards, guide dikes, jetties, riprap protection of abutments and embankments, embankment overflow at lower elevations than the bridge deck and alignment of piers with the flow.

32-6.11 Waterway Enlargement

There are situations where roadway and structural constraints dictate the vertical positioning of a bridge that results in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases.

It is possible to increase the effective area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. There are, however, several factors that must be accommodated when this action is taken.

1. The flow line of the flood channel should be set above the stage elevation of the dominant discharge. See the AASHTO *Highway Drainage Guidelines*.

2. The flood channel must extend far enough up and downstream of the bridge to establish the desired flow regime through the affected reach.

3. The flood channel must be stabilized to prevent erosion and scour.

32-6.12 Auxiliary Openings

The need for auxiliary waterway openings, or relief openings as they are commonly termed, arises on streams with wide floodplains. The purpose of openings on the floodplain is to pass a portion of the flood flow in the floodplain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but it has predictable capacity during flood events.

Basic objectives in choosing the location of auxiliary openings include the following:

1. maintenance of flow distribution and flow patterns,
2. accommodation of relatively large flow concentrations on the floodplain,
3. avoidance of floodplain flow along the roadway embankment for long distances, and
4. crossing of significant tributary channels.

The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The development of 2-D models, such as FESWMS, is a major step toward more adequate analysis of complex stream-crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between the two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of auxiliary openings should usually be generous to guard against that possibility.

32-7.0 DESIGN PROCEDURE FORM

A checklist to document the design procedures, studies, decisions, criteria, calculations, etc., for the bridge hydraulics is shown as Figure 32-7A, Design Procedure Checklist.

32-8.0 REFERENCES

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